SEISMIC DESIGN OF ELEVATED TANKS

A PROJECT REPORT SUBMITTED IN PARTIAL FULFILLMENT
OF THE REQUIREMENTS FOR THE DEGREE OF

BACHELOR OF TECHNOLOGY
IN
CIVIL ENGINEERING

BY

MANORANJAN SAHOO
(Roll Nos. 10301017)
AND
TANDRITA BISWAS
(Roll No. 10301020)

DEPARTMENT OF CIVIL ENGINEERING
NATIONAL INSTITUTE OF TECHNOLOGY
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UNDER THE GUIDANCE OF
PROF. B.K. RATH

DEPARTMENT OF CIVIL ENGINEERING
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CERTIFICATE

This is to certify that the Project Report entitled "**Seismic Design Of Elevated Tanks**" submitted by **mr. manoranjan sahoo** and **miss tandrita biswas** in partial fulfillment of the requirements for the award of Bachelor Of Technology Degree in **Civil Engineering** at the National Institute Of Technology, Rourkela (Deemed University) is an authentic work carried out by them under my supervision and guidance.

To the best of my knowledge, the matter embodied in this Project Report has not been submitted to any other University/Institute for the award of any Degree or Diploma.

Date:

Prof. B.K Rath
Department Of Civil Engineering
National Institute Of Technology
Rourkela-769008

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Lastly, we thank all those who are involved directly or indirectly in completion of the present project work.

TANDRITA BISWAS
Roll No. 10301020

MANORANJAN SAHOO
Roll No. 10301017

CONTENTS

SEISMIC DESIGN OF ELEVATED TANKS

MANORANJAN SAHOO and TANDRITA BISWAS

SUPERVISION: Prof. B.K. Rath

ABSTRACT

In this study, Wind Force and Seismic forces acting on an Elevated water tank e.g. Intze Tank are studied. Seismic forces acting on the tank are also calculated changing the Seismic Response Reduction Factor(R). IS: 1893-1984/2002 for seismic design and IS: 875-1987(Part III) for wind load has been referred. Then checked the Design of Intze Tank by using the software STAAD PRO.

An Earthquake is a phenomenon that results from and is powered by the sudden release of stored energy in the crust that propagates Seismic waves. At the Earth's surface, earthquakes may manifest themselves by a shaking or displacement of the ground and sometimes tsunamis, which may lead to loss of life and destruction of property.Seismic safety of liquid tanks is of considerable importance. Water storage tanks should remain functional in the post earthquake period to ensure potable water supply to earthquake-affected regions and to cater the need for fire fighting demand. Industrial liquid containing tanks may contain highly toxic and inflammable liquids and these tanks should not loose their contents during the earthquake. The current design of supporting structures of elevated water tanks are extremely vulnerable under lateral forces due to an earthquake as it is designed only for the wind forces but not the seismic forces.

The strength analysis of a few damaged shaft types of stagings clearly shows that all of them either met or exceeded the strength requirement of IS: 1893-1984 however they were all found deficient when compared with requirements of International Building Codes.Frame type stagings are generally regarded superior to shaft type of stagings for lateral resistance because of their large redundancy and greater capacity to absorb seismic energy through inelastic actions.

Various Codes have been considered and the maximum value of the ratio of base shear coefficient of tank to building, ($BSC_{\mathbf{tank}}$ / $BSC_{\mathbf{bldg}}$) is about 3 to 4 in all the codes, as against a value of 6 to 7 for low ductility tanks. This implies that design base shear for a low ductility tank is double that of a high ductility tank.

Indian Standard IS: 1893-1984 provides guidelines for earthquake resistant design of several types of structures including liquid storage tanks. This standard is under revision and in the revised form it has been divided into five parts. First part, IS 1893 (Part 1): 2002; which deals with general guidelines and provisions for buildings has already been published. Second part, yet to be published, will deal with the provisions for liquid storage tanks. In this section, provisions of IS: 1893-1984 for buildings and tanks are reviewed briefly followed by an outline of the changes made in IS 1893 (Part 1): 2002.

Any design of water tanks is subjected to Dead Load + Live Load and Wind Load or Earthquake load as per I S Code of Practice. Most of the times tanks are designed for Wind Load and not even checked for Earthquake load assuming that the tanks will be safe under Earthquake Loads once designed for Wind Loads. However present observation on the earthquake at Bhuj has shown that this tanks must have been designed for Wind Loads but did not stand Earthquake Load. Keeping this in view two Intze Tanks are designed with different specifications are studied by taking into account the provisions of 1893:2002 and for Elevated Tank 1893:1984 as well as NICEE suggestions and the results are presented.

We have concluded that there is no uniformity in type of tanks described in various documents. All documents suggest consideration of Convective and Impulsive Components in seismic analysis of tanks. Ratio of Base Shear of tank and building is 6 to 7 for low ductility tanks and 3 to 4 for high ductility tanks. Suitable provisions for lower bound limit on spectral values for tanks are necessary. Indian Code needs to include provisions on lower bound limit on spectral values of buildings and tanks and also Convective Mode of vibration in the seismic analysis of tanks. Based on the review of various International Codes, it is recommended that IS 1893 should have values of R in range of 1.1 to 2.25 for different types of tanks. R Value taken in IS 1893:1984 is nowhere in the range corresponding to that value in different international codes. Base Shear and Base Moment increases from Zone 3 to Zone 4 to Zone 5. With the increase in R value Base Shear and Base Moment decreases. Considering the design aspect, the seismic forces remain constant in a particular Zone provided the soil properties remain same whereas the Wind force is predominant in coastal region, but in interior region earthquake forces are more predominant. For R= 2.25 and 1.8, column size (450 mm) and reinforcements (8,25 Φ bar) remain same but for R= 1.5, column size increases to 500 mm and reinforcements change to 8, 20 Φ bar. Using STAAD PRO also we got the same values.

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CHAPTER 1

INTRODUCTION

INTRODUCTION:

EARTHQUAKES:

An Earthquake is a phenomenon that results from and is powered by the sudden release of stored energy in the crust that propagates Seismic waves. At the Earth's surface, earthquakes may manifest themselves by a shaking or displacement of the ground and sometimes tsunamis, which may lead to loss of life and destruction of property.The word Earthquake is used to describe any seismic event—whether a natural phenomenon or an event caused by humans—that generates seismic waves.

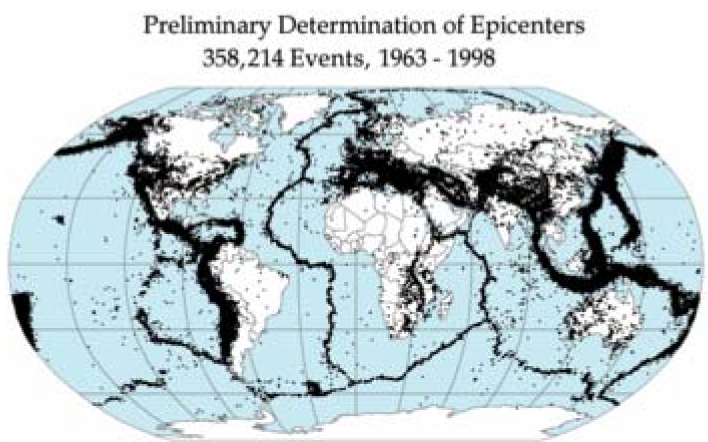


FIGURE 1.1

Most naturally occurring earthquakes are related to the tectonic nature of the earth. Such earthquakes are called tectonic earthquakes. The Earth's lithosphere is a patchwork of plates in slow but constant motion caused by the heat in the Earth's mantle and core. Plate boundaries grind past each other, creating frictional stress. When the frictional stress exceeds a critical value, called local strength, a sudden failure occurs. The boundary of tectonic plates along which failure occurs is called the fault plane. When the failure at the fault plane results in a violent displacement of the Earth's crust, the elastic strain energy is released and seismic waves are radiated, thus causing an earthquake.

Earthquakes occurring at boundaries of tectonic plates are called interplate earthquakes, while the less frequent events that occur in the interior of the lithospheric plates are called intraplate earthquakes.

The severity of an earthquake can be measured in terms of magnitude and intensity. For that seismologists use two fundamentally different but equally important types of scales. The original force or energy of an earthquake is measured on a magnitude scale. The Richter scale is a well known example of a magnitude scale. The second type of scale measures the intensity of shaking occurring at any given point on the Earth's surface. These scales are referred to as intensity scales. The Mercalli intensity scale, which measures the effects of the seismic waves, is an example of a commonly used intensity scale.

The non-specialized media will often refer to the magnitudes of earthquakes as being reported on the Richter scale. However, the magnitudes reported nowadays are actually on the moment magnitude scale. This is because the older Richter scale is not well-suited to accurately measure earthquakes with magnitudes over 6.8.

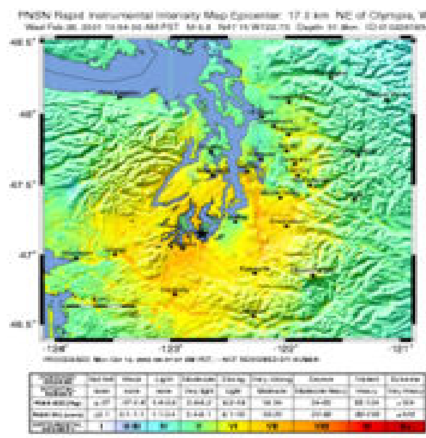
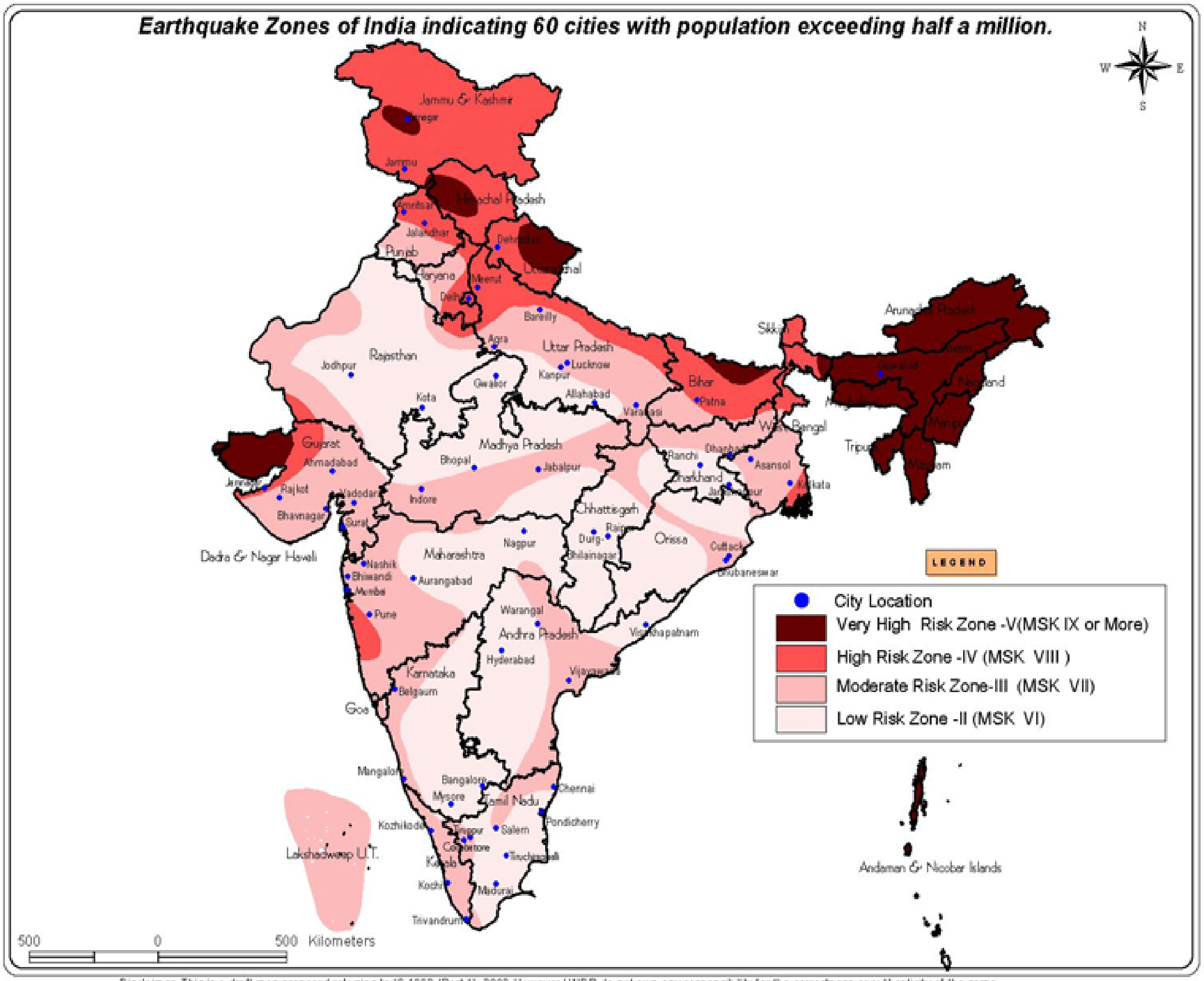


FIGURE 1.2: An isoseismal map

The 2001 Gujarat Earthquake centered near Bhuj rated 6.9 on the Richter Scale. It happened because of tectonic plates releasing pressure in the area after a collision margin took place. A collision margin is where two continental plates hit into each other and begin to rub creating lots of pressure until eventually it was released and it created an earthquake in Gujarat.

India is divided into different seismic zones. As per IS 1893:1984 Code India is divided from Zone 1 to Zone 5. But as per IS 1893:2002 Code it has been divided from Zone 2 to Zone 5. Zone 1 has been discarded.



Disclaimer: This is a draft map prepared referring to IS 1893 (Part 1): 2002. However UNDP do not own any responsibility for the correctness or authenticity of the same

FIGURE 1.3: Different Earthquake Zones of India

CHAPTER 2

PERFORMANCE OF ELEVATED TANKS

PERFORMANCE OF ELEVATED TANKS:

Seismic safety of liquid tanks is of considerable importance. Water storage tanks should remain functional in the post earthquake period to ensure potable water supply to earthquake-affected regions and to cater the need for fighting. Industrial liquid containing tanks may contain highly toxic and inflammable liquids and these tanks should not loose their contents during the earthquake. Liquid storage tanks are mainly of two types: ground supported tanks and elevated tanks. Elevated tanks are mainly used for water supply schemes and they could be supported on RCC shaft, RCC or steel frame, or masonry pedestal.

Two aspects came to forefront for the designing of tanks:

1) Due consideration should be given to sloshing effects of liquid and flexibility of container wall while evaluating the seismic forces on tanks.

2) It is recognized that tanks are less ductile and have low energy absorbing capacity and redundancy compared to the conventional building systems.

The current designs of supporting structures of elevated water tanks are extremely vulnerable under lateral forces due to an earthquake. The shaft type stagings suffer from poor ductility of thin shell sections besides low redundancy and toughness whereas framed stagings consist of weak members and poor brace columns joints. A strength analysis of a few damaged shaft type stagings clearly shows that all of them either met or exceeded the strength requirements of IS: 1893-1984, however they were all found deficient when compared with requirements of the International Building Code IS: 1893-1984 is unjustifiably low for these systems.

FIGURE 2.1: Collapsed 265 KL water tank in Chobari village about 20km from the epicenter of Bhuj earthquake. The tank was approximately half full during the earthquake.

FIGURE 2.2: Flexural cracks in staging of 500 kL tank being repaired by injecting epoxy. This tank in Morbi, 80km away from the epicenter, was empty at the time of the earthquake.

CHAPTER 3

FRAME TYPE STAGINGS

FRAME TYPE STAGINGS:

Frame type stagings are generally regarded superior to shaft type stagings for lateral resistance because of their large redundancy and greater capacity to absorb seismic energy through inelastic actions. Framed stagings have many flexural members in the form of braces and columns to resist lateral loads. RC frameworks can be designed to perform in a ductile fashion under lateral loads with greater reliability and confidence as opposed to thin shell sections of the shaft type staging. The sections near the beam-ends can be designed and detailed to sustain inelastic deformation and dissipate seismic energy. Frame members and the brace column joints are to be designed and detailed for inelastic deformations, or else a collapse of the staging may occur under seismic overloads. The collapse of the members could have been prevented if the members of stagings were detailed according to BIS.

FIGURE 3.1: Poor detailing of column-brace joints

However, if the frame members and the brace-column joints are not designed and detailed for inelastic deformations, a collapse of the staging may occur under seismic overloads. Termination of longitudinal bars in the joint region, 90 ˚ hooks for insufficient number of stirrups and poor quality of concrete are some obvious omissions leading to the failure of joints and eventually causing the collapse of the supporting frame. The collapse of the structure could have been prevented if the frame members of stagings were detailed according to provisions of IS: 13920-1993(BIS 1993a) and IS: 11682-1985 (BIS 1985) which refers to the ductility requirements of IS: 4326-1976(BIS 1976).

CHAPTER 4

COMPARISON OF DESIGN FORCES FROM VARIOUS CODES

COMPARISON OF DESIGN FORCES FROM VARIOUS CODES:

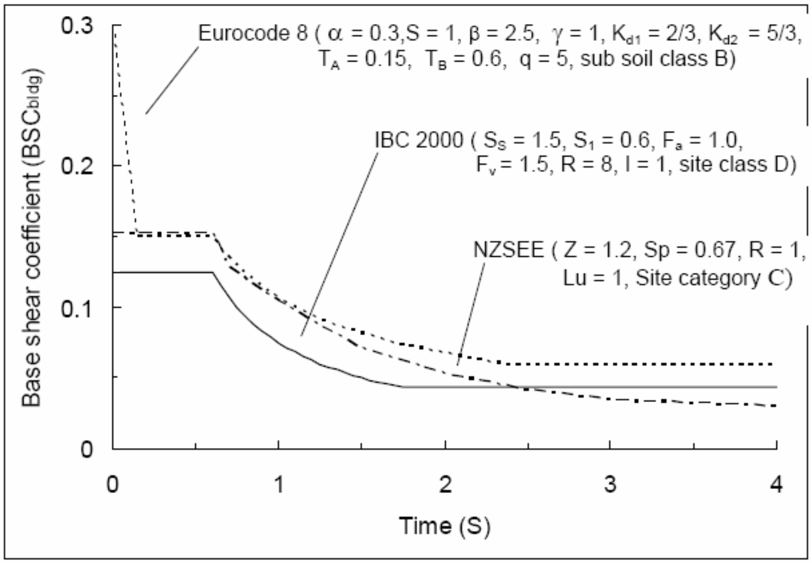


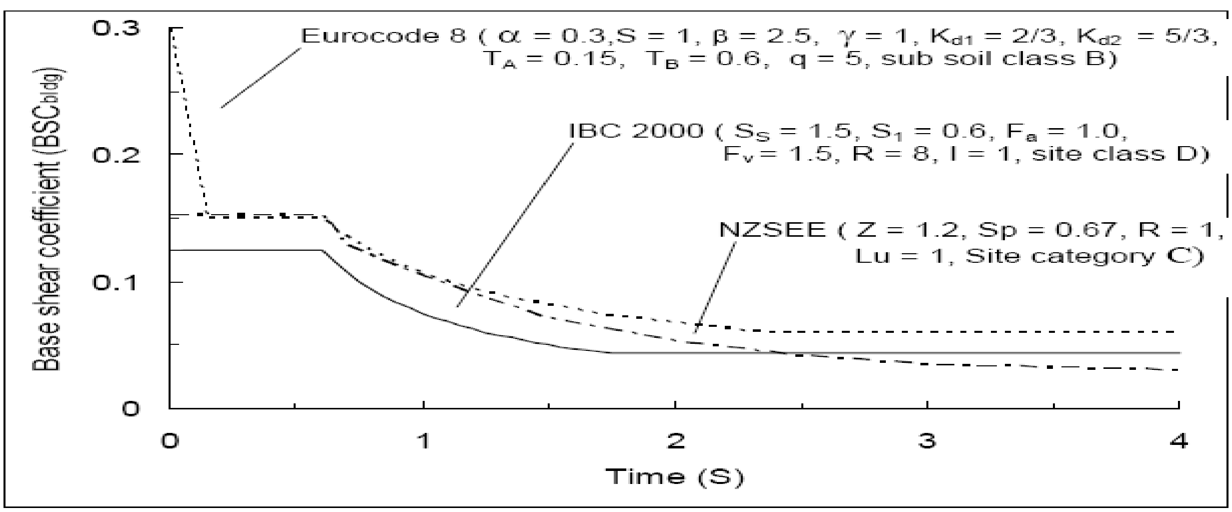
FIGURE 4.1: Comparison of base shear coefficient for ductile building obtained from various codes. Most severe zone in each code is considered

In figure 4.1, base shear coefficients of building (BSC $_{bldg}$), obtained from IBC 2000, Euro code 8 and NZS 4203:1992 are shown. These results correspond to the most severe zone of each code. It is seen that in the short period range (i.e., T=0.1-0.6s), results from Euro code 8 and NZS 4203 match well. In this short period range, IBC 2000 results are on lower side by about 15%. Further, all the three codes have different shape of spectra in constant-velocity range (i.e., T>0.6s). Moreover, magnitude of the lower bound limit on spectra is also seen to be different in these codes. To obtain similar comparison for tanks, first of all, for a particular type of tank, all the relevant parameters (such as R, q, C $_f$) from different codes will have to be identified. It is seen that most of the codes consider ground supported unanchored concrete water tank as a low ductility tank or a tank with low energy absorbing capacity. For such a tank the relevant parameters will be as shown in Table 4.1.

Code	Parameters
IBC 2000 and ACI 350.3	R = 1.5, I =1.25
Eurocode 8	q = 1.0, γ_I = 1.2
NZSEE guidelines	Sp = 1.0, μ = 1.25, ξ = 5%, C_f = 0.72

<u>TABLE 4.1: Parameters for a low ductility tank</u>

In Figure 4.2, comparison of base shear coefficient for this tank (BSC $_{tank}$) obtained from different codes is shown. It may be noted here that FEMA 368 has modified the base shear expressions of ACI 350.3 and AWWA D-110 and brought them in line with IBC 2000. In view of these modifications, parameters from ACI 350.3, AWWA D-110 and IBC 2000 are same. From Figure 4.2 it is seen that in the short period range (T<0.6s), Euro code 8 results are 10% higher and NZSEE results are 35% higher than the one obtained from IBC2000. Further, it can also be seen that except for IBC 2000, no other code has lower bound limit on base shear coefficient in long period range.



<u>FIGURE 4.2: Comparison of base shear coefficient for ground supported unanchored concrete water tank obtained from various codes. Most severe zone in each code is considered</u>

In figure 4.3 comparison of ratio of base shear coefficient of tank and building (BSC_{tank}/BSC_{bldg}) is shown. Here, base shear coefficient of tank from a particular code is divided by corresponding base shear coefficient of a ductile building. It is seen that from T=0.1-0.6s, this ratio is constant for all the codes. This constant value is 6 for Euro code 8 and for IBC and NZSEE it is 6.7 and 7.3 respectively. The decrease in the value of this ratio for T>0.6s for the case of Euro code 8, is due to difference in shapes of spectrum used for tank and building. Another factor contributing to this decrease, particularly in higher period range, is absence of lower bound limit on spectral values for tanks. The decrease in the value of this ratio in long period range, for NZSEE, ACI 350.3 and AWWA D-110 is also attributed to similar reasons. For the case of IBC 2000, due to lower bound limit on spectral values for tanks, the ratio of tank to building shear does not fall below the value of 4, even in long period range. Results similar to one presented in Figure below, can be obtained for a high ductility tank, i.e., a tank with high-energy absorbing capacity. For such a tank, various parameters of different codes are given in Table 4.2. These parameters can as well be applicable to some of the elevated tanks. For Euro code 8, value of q = 2 is considered, which is suggested for a low risk category elevated tank with simple type of supporting structure.

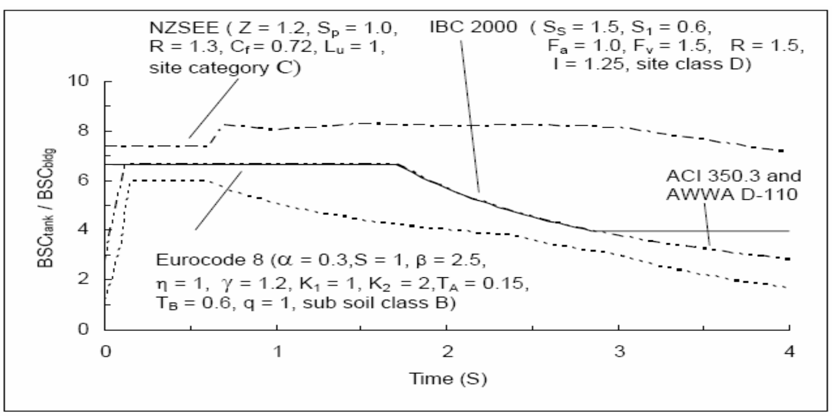


FIGURE 4.3: Comparison of ratio of base shear coefficients of tank and building from various codes (Low ductility tank).

Code	Parameters
IBC 2000 and ACI 350.3	R = 3.0, I =1.25
Eurocode 8	q = 2.0, γ_1 = 1.2
NZSEE guidelines	Sp = 1.0, μ = 3.0, ξ = 5%, C_f = 0.38

TABLE 4.2: Parameters for a high ductility tank

Results on ratio of base shear coefficient of tank to building, (BSC_{tank} / BSC_{bldg}), are shown in Figure 4.4. It is seen that maximum value of this ratio is about 3 to 4 in all the codes, as against a value of 6 to 7 for low ductility tanks. This implies that design base shear for a low ductility tank is double that of a high ductility tank. Variation in the ratio of base shear of tank and building, in the higher time period range is seen in figure 4.4.

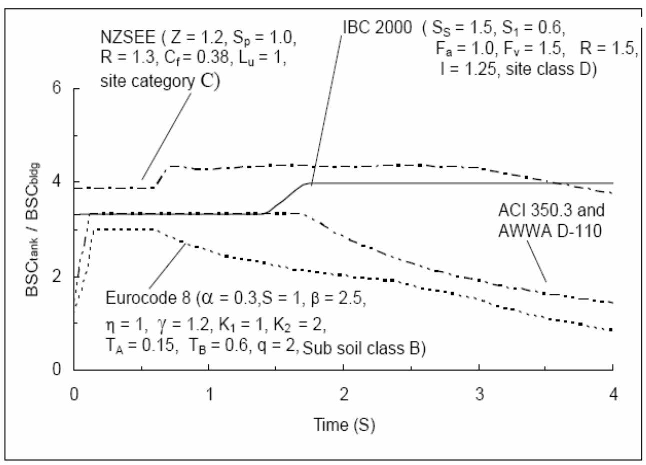


FIGURE 4.4: Comparison of ratio of base shear coefficient of tank and building from various codes (High ductility tank).

In figure 4.5 comparison of Base Shear Coefficient for convective mode for different codes is given. Results of Euro code 8 & API 650 match well whereas ACI 350.3 gives very high values of convective Base shear Coefficient. For time period greater than 3.0 seconds, the results of ACI 350.3 are 2.5 times higher than that of API 650.

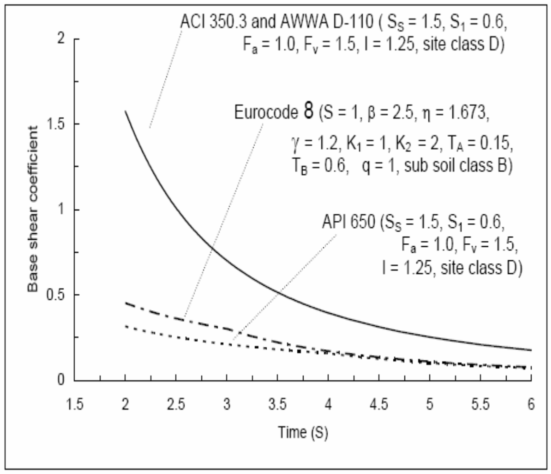


FIGURE 4.5: Comparison of base shear coefficient for convective mode

In figure 4.6 Base shear coefficient for low & high ductility tanks from IBC 2000 is given & it is found that the values obtained from IS 1893-1984 are on lower side. To achieve this level of BSC the value of R in IS 1893(Part 2): 2002 should be as given in Table 1.

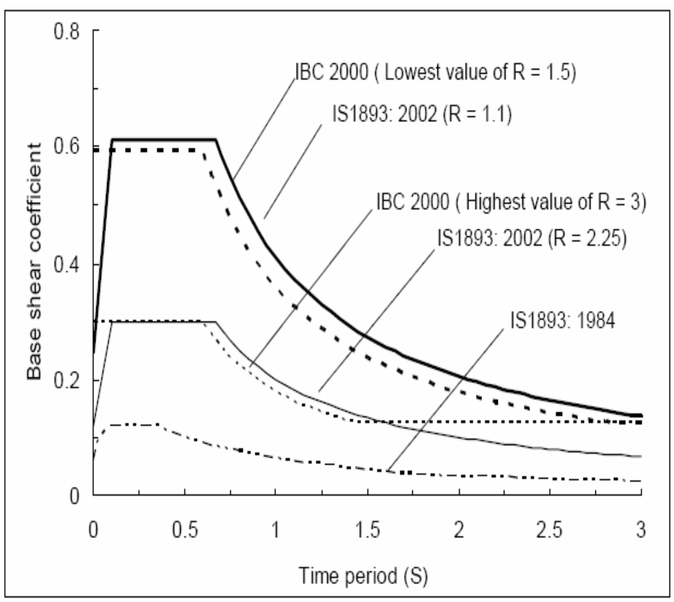


FIGURE 4.6: Base shear coefficients for tanks from IBC 2000, IS 1893:1984 and IS 1893(Part1): 2002

CHAPTER 5

PROVISIONS OF INDIAN CODE

PROVISIONS OF INDIAN CODE:

Indian Standard IS: 1893-1984 provides guidelines for earthquake resistant design of several types of structures including liquid storage tanks. This standard is under revision and in the revised form it has been divided into five parts. First part, IS 1893 (Part 1): 2002; which deals with general guidelines and provisions for buildings has already been published. Second part, yet to be published, will deal with the provisions for liquid storage tanks. In this section, provisions of IS: 1893-1984 for buildings and tanks are reviewed briefly followed by an outline of the changes made in IS 1893 (Part 1): 2002.

In IS: 1893-1984, Base Shear for building is given by $V = C_s W$, where, C_s is the Base Shear Coefficient given by

$C_s = K\,C\,\beta I\,\alpha_o$.

Here,

K = Performance factor depending on the structural framing system and brittleness or ductility of construction;

C = Coefficient defining flexibility of structure depending on natural period T;

β = Coefficient depending upon the soil-foundation system;

I = Importance factor;

α_o = Basic Seismic Coefficient depending on Zone.

For buildings with moment resisting frames, K = 1.0. Importance factor, for buildings is usually I = 1.0.

IS: 1893-1984; does not have any provision for ground-supported tanks. It has provisions for elevated tanks, for which it does not consider Convective Mode. Base Shear for elevated tank is given by $V = C_s W$, where, Base Shear Coefficient, C_s is given by

$$C_s = \beta I F_o (S_a/g)$$

Here,

S_a/g = Average Acceleration Coefficient corresponding to the time period of the tank, obtained from acceleration spectra given in the code;

F_o = Seismic Zone Factor;

W = Weight of container along with its content and one-third weight of supporting structure.

For elevated tanks, Importance factor I = 1.5. It may be noted that in the expression for Base Shear Coefficient of tank, the Performance Factor K does not appear, i.e. K = 1 is considered, which is same as that for a building with ductile frame. This implies that in IS: 1893-1984, there is no provision to account for lower ductility and energy absorbing capacity of elevated tanks. Thus, as per IS: 1893-1984, Base Shear Coefficient for tank will be only 1.5 times higher than that for a building, which is due to higher value of Importance Factor. This is in contrast to other codes, reviewed in earlier sections, wherein tank Base Shear Coefficient is seen to be 3 to 7 times higher than buildings. This lacunae needs to be corrected in the next revision of the code.

As mentioned earlier, IS 1893 is under revision and first part, of the revised code, IS 1893 (Part 1): 2002, has already been published. In this revised code, Base Shear for building is given by $V = C_s W$, and base shear coefficient C_s is given by

$$C_s = ZI (S_a/g)/2R$$

Where

Z = Zone Factor,

I = Importance Factor,

R = Response Reduction Factor and

S_a/g = Average Response Acceleration Coefficient, obtained from acceleration spectra given in the code.

For buildings with ductile frames value of R is 5.

In Figure 5.1, a comparison of Base Shear Coefficients for building obtained from IS: 1893-1984 and IS 1893 (Part I): 2002 is shown, along with the Base Shear Coefficient from IBC 2000.

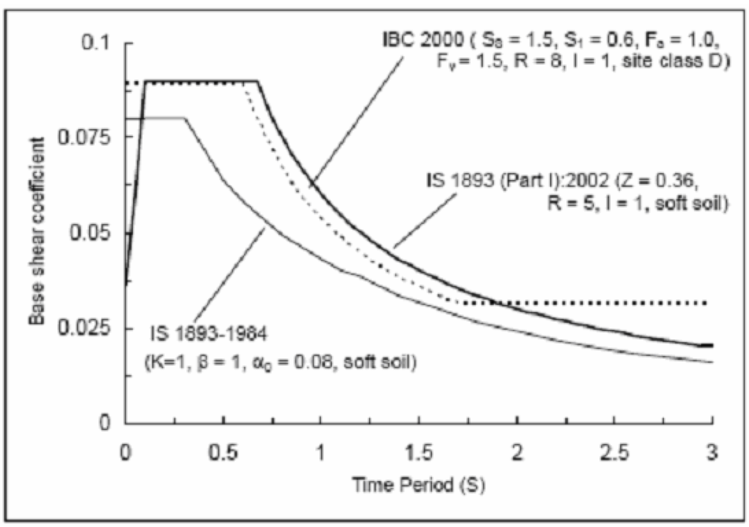


FIGURE 5.1: Comparison of BSC of Building obtained from IS Codes & IBC 2000

Since IS: 1893-1984, does not specify specific value of load factors for strength design, the results in Figure above are presented for working stress level. It is seen that Base Shear Coefficient from IS: 1893-1984 is lower than one from IS 1893 (Part I): 2002. Further, unlike IBC 2000, there is no lower bound in IS 1893 (Part I): 2002 and IS: 1893-1984.

Subsequent parts of IS 1893:2002, will be using Acceleration Spectra given in Part I, and will be based on same design philosophy. Thus, for liquid storage tanks, Base Shear Coefficient will be given by $C_s = ZI (S_a/g)/2R$ in which suitable values of R will have to be used for different types of tanks. From the review presented in earlier sections, it is seen that low and high ductility tanks have design Base Shear 3 to 7 times higher than ductile buildings.

In Figure5.2, base shear coefficients for low and high ductility tanks, from IBC 2000 (i.e., tanks with R=1.5 and R=3.0) are shown. To achieve this level of base shear coefficients the value of R in IS 1893 (Part 1): 2002 should be 1.1 and 2.25 as can be seen from Figure below. Also shown in this figure is the base shear coefficient for tank obtained from IS: 1893-1984, which is on much lower side. Based on the comparison shown in the Figure 5.2, proposed values of R, which can be used in, IS 1893 (Part 2): 2002 for different types of tanks are given in Table 3.

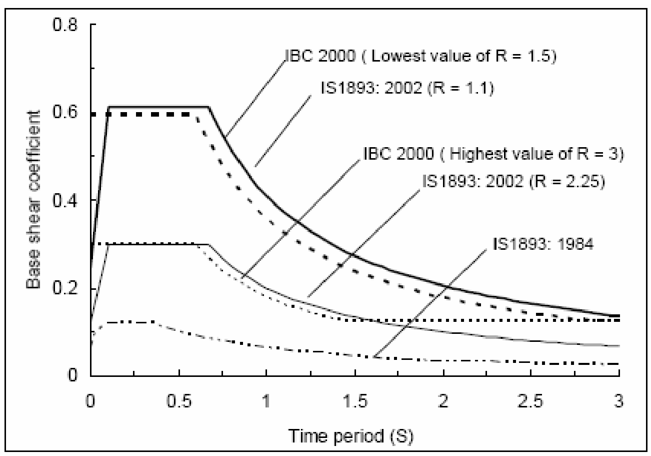


FIGURE 5.2: Base Shear Coefficients for tanks from IBC 2000, IS 1893:1984 and IS 1893(Part 1): 2002. IBC values are divided by 1.4 to bring them to working stress level.

Tank	Proposed value of R
Ground supported tanks	
Unanchored steel tank	1.80
Anchored steel tank	2.25
Concrete tank with unconstrained flexible base	1.10
Concrete tank with non-sliding base	1.50
Elevated tanks	
Supported on RCC shaft	1.50
Supported on RCC frame staging	2.25
Supported on steel frame staging	2.25
Supported on masonry shaft	1.10

TABLE 5.1:Proposed values of Response Reduction Factor, R for IS 1893:2002

CHAPTER 6

DESIGN OF INTZE TANK

DESIGN OF INTZE TANK

Any design of water tanks is subjected to Dead Load + Live Load and Wind Load or Earthquake load as per I S Code of Practice. Most of the times tanks are designed for Wind Load and not even checked for Earthquake load assuming that the tanks will be safe under Earthquake Loads once designed for Wind Loads. However present observation on the earthquake at Bhuj has shown that these tanks must have been designed for Wind Loads but did not stand Earthquake Load. Keeping this in view two Intze Tanks with different specifications are studied as follows by taking into account the provisions of 1893:2002 and for Elevated Tank 1893:1984 as well as NICEE suggestions.

DESIGN OF INTZE TANK CONSIDERING SEISMIC FORCE

6.1 PROBLEM STATEMENT: -

Design a water tank of 600,000 litres. The height of tank is 12m up to the bottom of tank.

SOLUTION: -

- Concrete Mix is of M20.
- 12m diameter is adopted for cylindrical portion. For this portion height adopted is 5m.
- Free Board 20cm is provided.
- Radius of Bottom Dome= 5.8m.

Design of Roof dome:

- 10cm thick Roof Dome is provided.
- Total Load=3.9 KN/m^2
- Radius of Dome is 10m.
- Maximum Hoop Stress=1.95 Kg/cm^2

- Nominal Steel of 8mm Φ bars @ 200mm c/c are provided both ways.
- Meridonial Stress= 2.167 Kg/cm^2

Design of Ring Beam:

- Horizontal Thrust/cm length= 17.336 Kg/cm^2.
- Hoop Tension= 10401.6 Kg.
- Tensile Stress= 10.9 Kg/cm^2.
- Ring Beam is 30cmx30cm.
- 4,12mm Φ bars and 6mm Φ stirrups @ 30cm c/c are provided.

Design of Cylindrical Wall:

- Hoop Tension= 300000 N
- Wall thickness is 250mm thick at base and 150mm at top.
- 10,12mm Φ bars @ 10cm c/c and 8mm Φ bars @ 150mm c/c on both faces.

Design of Ring Beam at junction of cylindrical wall and conical wall:

- Total Load= 47680 N
- Meridonial Thrust in the Conical Dome= 67429.7 N
- Total Hoop Tension= 466080 N
- Tensile Stress= 1.19<1.2

Design of Conical Dome:

- Total Vertical Load= 6,271,353.65 N
- Meridonial Stress= 1.008 N/mm^2
- Thickness of Conical Dome= 350mm.
- Provide 20mm Φ bars on one side and 10mm Φ bars @ 8mm c/c.
- Distribution Steel= 8mm Φ bars @ 12mm c/c

Design of Bottom Dome:

- Radius of Bottom Dome = 5.8 m
- 20 cm thickness is provided.
- Total Load= 54120 Kg
- Meridonial Stress= 9.1 Kg/cm^2
- Hoop Stress= 7.84 Kg/cm^2
- 8mm Φ bars @ 12cm centers both ways.

Design of Circular Beam:

- Horizontal Thrust on circular beam= 10860 Kg/m
- Vertical load on beam /m= 36580 Kg/m
- Section of Beam is 100cmx50cm
- Beam will be supported on 8 columns
- Maximum Bending Moment (-ve)= 31330Kgm
- Effective Depth= 66.6 cm
- Over all Depth= 100cm
- Provide 6 bars of 20mm Φ at centre and 5, 16mm Φ at support
- Shear Reinforcement: Provide 12 mm Φ, 6 legged stirrups @ 9cm c/c at support.
- Shear Reinforcement: Provide 12mm Φ, 4 legged stirrups @ 9cm c/c at centre.
- Longitudinal Steel: Provide 8 bars of 12mm Φ, 4 cm each face.

Design of Column:

- Total vertical load on column: 1244310 N
- 50 cm diameter column is provided
- Provide 6 bars of 20 mm Φ and 6 mm Φ ties at 25 cm c/c.

6.2 PROBLEM STATEMENT: -

Design an Intze Tank of capacity150m^3. Safe Bearing Capacity of soil=12t/m^2.Seismic Zone III. Use M250 Concrete Mix and freeboard 30cm.

SOLUTION: -

Radius Of Bottom Dome:

- R=4.698 m

Design Of Roof Dome:

- Thickness= 10 cm
- Radius Of Dome= 12.259 m
- Nominal Steel of 12mm Φ bars @ 20 cm c/c are provided both ways.

Design Of Ring Beam At Top:

- Ring Beam Of Depth 300 cm and 400 mm width.
- Provide 5 bars of 6 mm Φ.
- Tie the 10 mm Φ bars by 6 mm dia nominal stirrup @ 200 mm c/c.

Design Of Cylindrical Wall:

- Thickness of wall= 150 mm
- Provide 12 mm Φ bars @180 mm.
- Provide 8 mm Φ bars @ 200 mm c/c on both face.

Design Of Ring Beam at Junction Of Cylindrical Wall And Conical Wall:

- 1000 mm x 150 mm Ring Beam is Provided.
- Provide 12 mm Φ Bars @ 130 mm c/c.
- Provide 8 mm Φ stirrups @ 200 mm c/c.

Design Of Conical Dome:

- Thickness of 17 cm for Comical Dome.
- Provide 12 mm Φ bars spacing @ 100 mm c/c.
- Provide 8 mm Φ bars @ 200 mm c/c

Design Of Bottom Dome:

- Radius= 4698 mm
- Thickness Of Dome is 200 mm.
- Provide 8 mm Φ bars @ 150 mm c/c.

Design Of Circular Beam:

- 850 mm x 400 mm Section is taken.
- Radius= 3.240 m
- Provide 850 mm Depth.
- Provide 7 bars of 25 mm Φ.
- Provide 5 bars of 20 mm Φ.
- Provide 12 mm Φ, 6 legged stirrups @ 100 mm as Shear Reinforcement.
- Provide 5 bars of 20 mm Φ and 8 bars of 12 mm Φ on two faces.

Design Of Columns:

- Tank is supported on 6 columns.
- Height Of Staging= 20 m.
- 8 bars of 16 mm Φ @ 250 mm c/c.
- In 450 mm column size Provide 25 mm Φ bar.

Design Of Braces:

- 7 Bars of 314 mm Φ both at Top and Bottom.
- Shear Reinforcement is 8 mm Φ @ 150 mm c/c.

If Tank will be at Rourkela:

- Wind Speed: 39 m/s
- Provide 20 mm Φ bars

ZONE	BASE SHEAR (N)	BASE MOMENT (KNM)
3	64508	1088
4	96758.7	1633.008
5	145138.1	2449.512

TABLE 6.1: Comparison Of Base Shear And Moment In Different Zone

R	BASE SHEAR (N)	BASE MOMENT (KNM)	REINFORCEMENTS	COLUMN SIZE
2.25	56826.58	959.068	8,25 Φ bar	450 mm
1.8	70649.2	1192.3	8,25 Φ bar	450 mm
1.5	83475.2	1451.562	8,20 Φ bar	500 mm

TABLE 6.2: Comparison Of Base Shear and moment with respect to R value

BHUBANESHWAR	ROURKELA
BASE SHEAR	**BASE SHEAR**
86310.6 N	63868.8 N
BASE MOMENT	**BASE MOMENT**
1237.361 KNM	1008.879 KNM

TABLE 6.3: Base Shear and Base Moment due to Wind Force

BHUBANESHWAR	ROURKELA
BASE SHEAR	**BASE SHEAR**
64508 N	64508 N
BASE MOMENT	**BASE MOMENT**
1088 KNM	1088 KNM

TABLE 6.4: Base Shear and Base Moment due to Seismic Force

<u>After analysis of the tank we observe that</u>: -

- Hydrodynamic forces exerted by liquid on tank shall be considered in addition to hydrostatic forces.
- But container of tank, which is designed by working stress method, when earthquake forces are considered, permissible stresses are increased by 33%. Hence hydrodynamic pressure in this case does not affect the container design.
- The moment and shear forces calculated due to earthquake load are more than the moment and shear forces due to wind load for staging.
- Structure of high importance like water tank should be designed according to earthquake load. Hence the available design is inadequate to sustain the earthquake.

CHAPTER 7

DISCUSSIONS and CONCLUSIONS

DISCUSSIONS:

- In India elevated tanks are widely used and these tanks have various types of supports.
- IS Code not yet specified R for water tanks however a proposal has been given by NICEE as shown in Table 4.2. It is felt that a detailed investigation is needed to ascertain their energy absorbing capacity and ductility characteristics.
- Suitable value of lower bound limits on spectral values for structures including tanks needs to be arrived at.
- AWWA D-100 does not recommend consideration of Convective Mode of vibration but Euro Code 8 and NZSEE does recommend.
- IBC 2000 and ACI 371 suggest that Convective Mode need not be considered if certain conditions on weight of water and time period of Convective Mode are met with.
- R Value taken in IS 1893:1984 is nowhere in the range corresponding to that value in different international codes.
- As per observed from Table 6.1, Base Shear and Base Moment have increased from Zone 3 to Zone 4 by almost 50 % and from Zone 4 to Zone 5 it has increased by another 50%.
- From Table 6.2 we observe the following:

CHANGE IN VALUE OF R	**% DECREASE IN BASE SHEAR**	**% DECREASE IN BASE MOMENT**
1.5 to 1.8	15	18
1.8 to 2.25	19	20

- From Table 6.3 we observe that due to change in place from Bhubaneshwar to Rourkela Base Shear due to Wind Force decreases by 26% and Base Moment decreases by 18%.

- Similarly from Table 6.4 we observe that due to change in place from Bhubaneshwar to Rourkela, lying in same Zone, Base Shear and Base Moment due to Seismic Force remains constant.
- We also observe that in Bhubaneshwar (coastal area) the Base Shear and Base Moment due to Wind Force is more by 25% and 12 % respectively as compared to Base Shear and Base Moment due to Seismic Force.
- In Rourkela (interior) the Base Shear due to Wind Force is less by 1% as compared to Base Shear due to Seismic Force and Base Moment is less by 7.8% almost remains constant.
- For R= 2.25 and 1.8, column size (450 mm) and reinforcements (8,25 Φ bar) remain same but for R= 1.5, column size increases to 500 mm and reinforcements change to 8, 20 Φ bar. Using STAAD PRO also we got the same values.

CONCLUSIONS:

The following conclusions have been drawn from the comparative assessment of provisions of different Code: -

- There is no uniformity in type of tanks described in various documents.
- All documents suggest consideration of Convective and Impulsive Components in seismic analysis of tanks.
- Ratio of Base Shear of tank and building is 6 to 7 for low ductility tanks and 3 to 4 for high ductility tanks.
- Most of the documents don't provide lower bound limit on spectral values for tanks.
- Suitable provisions for lower bound limit on spectral values for tanks are necessary. Only ACI 371, dealing with elevated tanks and IBC 2000 have such provisions.
- Convective Mode Base Shear values obtained from API 650 and Euro Code 8 are similar but those obtained from ACI 350.3 is 2.5 times greater than that of ACI 370.
- Few inconsistencies among different AWWA standards need to be resolved.
- Indian Code needs to include provisions on lower bound limit on spectral values of buildings and tanks and also Convective Mode of vibration in the seismic analysis of tanks.
- Based on the review of various International Codes, it is recommended that IS 1893 should have values of R in range of 1.1 to 2.25 for different types of tanks.
- R Value taken in IS 1893:1984 is nowhere in the range corresponding to that value in different international codes.
- Base Shear and Base Moment increases from Zone 3 to Zone 4 to Zone 5.
- With the increase in R value Base Shear and Base Moment decreases.
- The seismic forces remain constant in a particular Zone provided the soil properties remain same.

- In coastal region Wind force is predominant, but in interior earthquake forces are more predominant lying in the same Zone.
- For R= 2.25 and 1.8, column size (450 mm) and reinforcements (8,25 Φ bar) remain same but for R= 1.5, column size increases to 500 mm and reinforcements change to 8, 20 Φ bar. Using STAAD PRO also we got the same values.

REFERENCES

1. Rai Durgesh C; "Performance of Elevated Tanks in Bhuj Earthquake"; Proc. Indian Acad. Sci. (Earth Planet Sci.), **112**, No. 3, September 2003.pp 421-429.

2. Jaiswal O. R., Rai Durgesh C and Jain Sudhir K; "Review of Code Provisions on Design Seismic Forces for Liquid Storage Tanks"; Document No.: IITK-GSDMA-EQ01-V1.0, Final Report: A - Earthquake Codes, IITK-GSDMA Project on Building Codes.

3. Indian Institute of Technology Kanpur, IITK GSDMA Guidelines for Seismic Design of Liquid Storage Tanks, August 2005.

4. IS 1893:1984, CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES

5. IS 1893(Part I): 2002, CRITERIA FOR EARTHQUAKE RESISTANT DESIGN OF STRUCTURES (**PART 1 GENERAL PROVISIONS AND BUILDINGS)**

6. IS 875:1987, Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures Part 3: Wind Loads

7. Vazirani & Ratwani, "Concrete Structures", Khanna Publishers, Year of Publication 1996

APPENDIX

DESIGN USING STAAD PRO

```
**************************************************
*                                       *
*          STAAD.Pro                    *
*          Version  2004    Bld 1001.INDIA       *
*          Proprietary Program of              *
*          Research Engineers,  Intl.          *
*          Date=    MAY  1, 2007               *
*          Time=    23:13:29                 *
*                                       *
*     USER ID: Snow Panther [LZ0]              *
**************************************************
```

```
INPUT FILE: Structure1.STD
     1. STAAD SPACE
     2. START JOB INFORMATION
     3. ENGINEER DATE 03-APR-07
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT METER KN
     7. JOINT COORDINATES
     8. 1 3 3 0; 3 3.2325 3 0.402702; 5 3.6975 3 0.402702; 7 3.93 3 0
     9. 9 3.6975 3 -0.402702; 11 3.2325 3 -0.402702; 12 3 -20 0; 13 3.233 -20 0.403
    10. 14 3.697 -20 0.403; 15 3.93 -20 0; 16 3.697 -20 -0.403; 17 3.232 -20 -0.403
    11. MEMBER INCIDENCES
    12. 4 1 12; 5 3 13; 6 5 14; 7 7 15; 8 9 16; 9 11 17
    13. DEFINE MATERIAL START
    14. ISOTROPIC CONCRETE
    15. E 2.17185E+007
    16. POISSON 0.17
    17. DENSITY 23.5616
    18. ALPHA 1E-005
    19. DAMP 0.05
    20. END DEFINE MATERIAL
    21. MEMBER PROPERTY AMERICAN
    22. 4 TO 9 PRIS YD 0.45
    23. CONSTANTS
    24. MATERIAL CONCRETE MEMB 4 TO 9
    25. SUPPORTS
    26. 12 TO 17 FIXED
    27. DEFINE UBC LOAD
   **WARNING- THIS STRUCTURE IS DISJOINTED. IGNORE IF
          MASTER/SLAVE OR IF UNCONNECTED JOINTS.
    28. ZONE 0.16 I 1.5 RWX 9 RWZ 9 CT 0.032 S 1.5
    29. SELFWEIGHT
    30. JOINT WEIGHT
    31. 1 3 5 7 9 11 WEIGHT 56.826
```

32. LOAD 2
33. SELFWEIGHT Y -1
34. JOINT LOAD
35. 1 3 5 7 9 11 FY -2.2
36. LOAD 5 MOMENT
37. JOINT LOAD
38. 12 TO 17 MY -959.068
39. PDELTA ANALYSIS PRINT LOAD DATA

STAAD SPACE

P R O B L E M   S T A T I S T I C S

NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS = 12/ 6/ 6
ORIGINAL/FINAL BAND-WIDTH= 6/ 1/ 6 DOF
TOTAL PRIMARY LOAD CASES = 2, TOTAL DEGREES OF FREEDOM = 36
SIZE OF STIFFNESS MATRIX = 1 DOUBLE KILO-WORDS
REQRD/AVAIL. DISK SPACE = 12.0/ 277.5 MB, EXMEM = 684.9 MB

LOADING 2

SELFWEIGHT Y -1.000

ACTUAL WEIGHT OF THE STRUCTURE = 517.129 KN

JOINT LOAD - UNIT KN METE

JOINT	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM-Z
1	0.00	-2.20	0.00	0.00	0.00	0.00
3	0.00	-2.20	0.00	0.00	0.00	0.00
5	0.00	-2.20	0.00	0.00	0.00	0.00
7	0.00	-2.20	0.00	0.00	0.00	0.00
9	0.00	-2.20	0.00	0.00	0.00	0.00
11	0.00	-2.20	0.00	0.00	0.00	0.00

LOADING 5 MOMENT

JOINT LOAD - UNIT KN METE

JOINT	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM-Z
12	0.00	0.00	0.00	0.00	-800.00	0.00
13	0.00	0.00	0.00	0.00	-800.00	0.00
14	0.00	0.00	0.00	0.00	-800.00	0.00

15	0.00	0.00	0.00	0.00	-800.00	0.00
16	0.00	0.00	0.00	0.00	-800.00	0.00
17	0.00	0.00	0.00	0.00	-800.00	0.00

++ Adjusting Displacements 23:13:29

************ END OF DATA FROM INTERNAL STORAGE ************

40. PRINT SUPPORT REACTION

SUPPORT REACTIONS -UNIT KN METE STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
12	2	0.00	88.39	0.00	0.00	0.00	0.00
	5	0.00	0.00	0.00	0.00	800.00	0.00
13	2	0.00	88.39	0.00	0.02	0.00	-0.03
	5	0.00	0.00	0.00	0.00	800.00	0.00
14	2	0.00	88.39	0.00	0.02	0.00	0.03
	5	0.00	0.00	0.00	0.00	800.00	0.00
15	2	0.00	88.39	0.00	0.00	0.00	0.00
	5	0.00	0.00	0.00	0.00	800.00	0.00
16	2	0.00	88.39	0.00	-0.02	0.00	0.03
	5	0.00	0.00	0.00	0.00	800.00	0.00
17	2	0.00	88.39	0.00	-0.02	0.00	0.03
	5	0.00	0.00	0.00	0.00	800.00	0.00

************** END OF LATEST ANALYSIS RESULT **************

41. PERFORM ANALYSIS PRINT STATICS CHECK

STATIC LOAD/REACTION/EQUILIBRIUM SUMMARY FOR CASE NO. 2

***TOTAL APPLIED LOAD (KN METE) SUMMARY (LOADING 2)
SUMMATION FORCE-X = 0.00
SUMMATION FORCE-Y = -530.33
SUMMATION FORCE-Z = 0.00

SUMMATION OF MOMENTS AROUND THE ORIGIN-
MX= 0.00 MY= 0.00 MZ= -1837.55

***TOTAL REACTION LOAD(KN METE) SUMMARY (LOADING 2)
SUMMATION FORCE-X = 0.00
SUMMATION FORCE-Y = 530.33
SUMMATION FORCE-Z = 0.00

SUMMATION OF MOMENTS AROUND THE ORIGIN-
MX= 0.00 MY= 0.00 MZ= 1837.55

MAXIMUM DISPLACEMENTS (CM /RADIANS) (LOADING 2)

	MAXIMUMS	AT NODE
X =	6.96262E-03	11
Y =	-3.01597E-02	11
Z =	4.14957E-03	11
RX=	2.42434E-06	11
RY=	-2.09864E-18	11
RZ=	4.06783E-06	3

STATIC LOAD/REACTION/EQUILIBRIUM SUMMARY FOR CASE NO. 5
MOMENT

***TOTAL APPLIED LOAD (KN METE) SUMMARY (LOADING 5)
SUMMATION FORCE-X = 0.00
SUMMATION FORCE-Y = 0.00
SUMMATION FORCE-Z = 0.00

SUMMATION OF MOMENTS AROUND THE ORIGIN-
MX= 0.00 MY= -4800.00 MZ= 0.00

***TOTAL REACTION LOAD(KN METE) SUMMARY (LOADING 5)

SUMMATION FORCE-X = 0.00
SUMMATION FORCE-Y = 0.00
SUMMATION FORCE-Z = 0.00

SUMMATION OF MOMENTS AROUND THE ORIGIN-
MX= 0.00 MY= 4800.00 MZ= 0.00

MAXIMUM DISPLACEMENTS (CM /RADIANS) (LOADING 5)

	MAXIMUMS	AT NODE
X =	0.00000E+00	0
Y =	0.00000E+00	0
Z =	0.00000E+00	0
RX=	0.00000E+00	0
RY=	0.00000E+00	0
RZ=	0.00000E+00	0

************ END OF DATA FROM INTERNAL STORAGE ************

42. START CONCRETE DESIGN
43. CODE IS13920
44. DESIGN COLUMN 4 TO 9

***NOTE: SOME OF THE BEAMS CONNECTED TO THE COLUMN NO. 4
ARE NOT DESIGNED. HENCE ONLY SHEAR FORCE FROM ANALYSIS
WILL BE CONSIDERED FOR SHEAR DESIGN.

==
=======

C O L U M N N O. 4 D E S I G N R E S U L T S

M30 Fe415 (Main) Fe415 (Sec.)

LENGTH: 23000.0 mm CROSS SECTION: 450.0 mm dia. COVER: 40.0 mm

** GUIDING LOAD CASE: 2 BRACED LONG COLUMN

REQD. STEEL AREA : 1823.25 Sq.mm.
MAIN REINFORCEMENT : Provide 8 - 25 dia. (1.19%, 1884.96 Sq.mm.)
(Equally distributed)
CONFINING REINFORCEMENT : Provide 8 mm dia. circular ties @ 100 mm c/c
over a length 3835.0 mm from each joint face towards
midspan as per Cl. 7.4.6 of IS-13920.
TIE REINFORCEMENT : Provide 8 mm dia. circular ties @ 225 mm c/c

SECTION CAPACITY (KNS-MET)

Puz : 2689.96 Muz1 : 114.72 Muy1 : 114.72

INTERACTION RATIO: 1.00 (as per Cl. 39.6, IS456:2000)

==
=======

***NOTE: SOME OF THE BEAMS CONNECTED TO THE COLUMN NO. 5
ARE NOT DESIGNED. HENCE ONLY SHEAR FORCE FROM ANALYSIS
WILL BE CONSIDERED FOR SHEAR DESIGN.

==
=======

C O L U M N N O. 5 D E S I G N R E S U L T S

M30 Fe415 (Main) Fe415 (Sec.)

LENGTH: 23000.0 mm CROSS SECTION: 450.0 mm dia. COVER: 40.0 mm

** GUIDING LOAD CASE: 2 BRACED LONG COLUMN

REQD. STEEL AREA : 1823.25 Sq.mm.
MAIN REINFORCEMENT : Provide 8 - 25 dia. (1.19%, 1884.96 Sq.mm.)
(Equally distributed)
CONFINING REINFORCEMENT : Provide 8 mm dia. circular ties @ 100 mm c/c
over a length 3835.0 mm from each joint face towards
midspan as per Cl. 7.4.6 of IS-13920.
TIE REINFORCEMENT : Provide 8 mm dia. circular ties @ 200 mm c/c

SECTION CAPACITY (KNS-MET)

Puz : 2689.96 Muz1 : 114.72 Muy1 : 114.72

INTERACTION RATIO: 1.00 (as per Cl. 39.6, IS456:2000)

===

***NOTE: SOME OF THE BEAMS CONNECTED TO THE COLUMN NO. 6
ARE NOT DESIGNED. HENCE ONLY SHEAR FORCE FROM ANALYSIS
WILL BE CONSIDERED FOR SHEAR DESIGN.

===

C O L U M N N O. 6 D E S I G N R E S U L T S

M30 Fe415 (Main) Fe415 (Sec.)

LENGTH: 23000.0 mm CROSS SECTION: 450.0 mm dia. COVER: 40.0 mm

** GUIDING LOAD CASE: 2 BRACED LONG COLUMN

REQD. STEEL AREA : 1823.25 Sq.mm.
MAIN REINFORCEMENT : Provide 8 - 25 dia. (1.19%, 1884.96 Sq.mm.)
(Equally distributed)
CONFINING REINFORCEMENT : Provide 8 mm dia. circular ties @ 100 mm c/c
over a length 3835.0 mm from each joint face towards
midspan as per Cl. 7.4.6 of IS-13920.
TIE REINFORCEMENT : Provide 8 mm dia. circular ties @ 200 mm c/c

SECTION CAPACITY (KNS-MET)

Puz : 2689.96 Muz1 : 114.72 Muy1 : 114.72

INTERACTION RATIO: 1.00 (as per Cl. 39.6, IS456:2000)

==
=======

***NOTE: SOME OF THE BEAMS CONNECTED TO THE COLUMN NO. 7
ARE NOT DESIGNED. HENCE ONLY SHEAR FORCE FROM ANALYSIS
WILL BE CONSIDERED FOR SHEAR DESIGN.

==
=======

C O L U M N N O. 7 D E S I G N R E S U L T S

M30 Fe415 (Main) Fe415 (Sec.)

LENGTH: 23000.0 mm CROSS SECTION: 450.0 mm dia. COVER: 40.0 mm

** GUIDING LOAD CASE: 2 BRACED LONG COLUMN

REQD. STEEL AREA : 1823.25 Sq.mm.
MAIN REINFORCEMENT : Provide 8 – 25 dia. (1.19%, 1884.96 Sq.mm.)
(Equally distributed)
CONFINING REINFORCEMENT : Provide 8 mm dia. circular ties @ 100 mm c/c
over a length 3835.0 mm from each joint face towards
midspan as per Cl. 7.4.6 of IS-13920.
TIE REINFORCEMENT : Provide 8 mm dia. circular ties @ 225 mm c/c

SECTION CAPACITY (KNS-MET)

Puz : 2689.96 Muz1 : 114.72 Muy1 : 114.72

INTERACTION RATIO: 1.00 (as per Cl. 39.6, IS456:2000)

==
=======

***NOTE: SOME OF THE BEAMS CONNECTED TO THE COLUMN NO. 8

ARE NOT DESIGNED. HENCE ONLY SHEAR FORCE FROM ANALYSIS
WILL BE CONSIDERED FOR SHEAR DESIGN.

===

C O L U M N   N O.   8 D E S I G N   R E S U L T S

M30   Fe415 (Main)   Fe415 (Sec.)

LENGTH: 23000.0 mm   CROSS SECTION: 450.0 mm dia. COVER: 40.0 mm

** GUIDING LOAD CASE: 2 BRACED LONG COLUMN

REQD. STEEL AREA   :   1823.25 Sq.mm.
MAIN REINFORCEMENT : Provide   8 – 25 dia. (1.19%,   1884.96 Sq.mm.)
(Equally distributed)
CONFINING REINFORCEMENT : Provide 8 mm dia. circular ties @ 100 mm c/c
over a length 3835.0 mm from each joint face towards
midspan as per Cl. 7.4.6 of IS-13920.
TIE REINFORCEMENT   : Provide 8 mm dia. circular ties @ 200 mm c/c

SECTION CAPACITY (KNS-MET)

Puz :   2689.96   Muz1 :   114.72   Muy1 :   114.72

INTERACTION RATIO: 1.00 (as per Cl. 39.6, IS456:2000)

===

***NOTE: SOME OF THE BEAMS CONNECTED TO THE COLUMN NO.   9
ARE NOT DESIGNED. HENCE ONLY SHEAR FORCE FROM ANALYSIS
WILL BE CONSIDERED FOR SHEAR DESIGN.

===

C O L U M N   N O.   9 D E S I G N   R E S U L T S

M30 Fe415 (Main) Fe415 (Sec.)

LENGTH: 23000.0 mm CROSS SECTION: 450.0 mm dia. COVER: 40.0 mm

** GUIDING LOAD CASE: 2 BRACED LONG COLUMN

REQD. STEEL AREA : 1823.25 Sq.mm.
MAIN REINFORCEMENT : Provide 8 – 25 dia. (1.19%, 1884.96 Sq.mm.)
(Equally distributed)
CONFINING REINFORCEMENT : Provide 8 mm dia. circular ties @ 100 mm c/c
over a length 3835.0 mm from each joint face towards
midspan as per Cl. 7.4.6 of IS-13920.
TIE REINFORCEMENT : Provide 8 mm dia. circular ties @ 200 mm c/c

SECTION CAPACITY (KNS-MET)

Puz : 2689.96 Muz1 : 114.72 Muy1 : 114.72

INTERACTION RATIO: 1.00 (as per Cl. 39.6, IS456:2000)

===

*******************END OF COLUMN DESIGN RESULTS*******************

45. END CONCRETE DESIGN
46. FINISH
STAAD SPACE -- PAGE NO. 13

*********** END OF THE STAAD.Pro RUN ***********

**** DATE= MAY 1,2007 TIME= 23:13:30 ****

****
* For questions on STAAD.Pro, *
* Please contact : Research Engineers Ltd. *
* E2/4,Block GP, Sector-V,Salt Lake, KOLKATA - 700 091 *
* India : TEL:(033)2357-3575 FAX:(033)2357-3467 *

*   email : support@calcutta.reiusa.com   *
*   US   : Ph-(714) 974-2500, Fax-(714) 921-0683   *
****